Aug 20th, 12:00 AM

Design and testing composite open web steel joists

J. A. Cran

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J. A. Cran, "Design and testing composite open web steel joists" (August 20, 1971). International Specialty Conference on Cold-Formed Steel Structures. Paper 3.
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Introduction

Traditionally, most buildings have been constructed in a utilitarian manner. The consideration of low first cost far outweighed the considerations of maintenance and future renovation costs. In the past decade, rapid technological change has spurred a dramatic shift in design emphasis. In order to avoid early obsolescence, designers are now concentrating on building flexibility.

For example, owners of schools and modern office buildings now demand the utmost in flexibility of interior design. Large open areas; long, column-free spans; moveable partitions; the ability to move heating, ventilating and air conditioning ducts; and ease in accommodating new electrical and communication requirements are mandatory. A most economical method of providing long clear spans, a high degree of services, and built-in flexibility of space utilization is the composite open web steel joist.

Advantages

The main advantages offered by composite open web steel joists (OWSJ) are as follows:

1. Economical long spans. Fig. 1 compares the cost per square foot of installed floor for various spans of composite and non-composite OWSJ. Fig. 1 is based on the following parameters:
   a) Joist depth - 32.0" 
   b) Joist spacing - 40'
   c) Steel floor - 1/4" deep, 22 ga., 6"c-c ribs
   d) Welded wire fabric - 6 x 6 10/10
   e) Concrete slab - 4" total thickness
   f) Stud shear connectors (composite joists only)
   g) Column and girder costs are not included.

   It can be seen that for spans of 37' and greater, composite open web steel joists offer cost savings. Below 40', the savings in OWSJ steel weight do not fully compensate for the added expense of shear connectors. Clear spans of 40' are commonplace in modern office towers while schools designed to an open plan require spans of 50-65'. Therefore, composite joists offer significant savings when used in schools and office towers.

2. Increased Natural Frequency. The natural frequency and amplitude of long span floor systems sometimes subject a building's occupants to objectionable vibrations. Composite joists have vibration characteristics superior to those of non-composite joists.

3. Increased Stiffness. The concrete-steel interaction results in composite joists having greater stiffness than non-composite joists. In practical terms, composite joists can be shallower than non-composite joists, or, for a given joist depth, live load deflections are significantly less when composite joists are used.

Shear Connection

As indicated when economy was discussed, the prime difference between a composite and non-composite joist is the presence of a shear connection. For both types of joists, the bottom chord and web systems are identical. However, because the presence of a shear connection in a composite joist allows the concrete floor slab to act with the steel joist, the top chord of a composite joist can be considerably lighter than that of a non-composite joist.

Because the material cost savings in the top chord must be sufficient to offset the added cost of a shear connection, it is desirable to have an economical connection. To this end, Stelco conducted a series of push-out tests to evaluate various types of shear connectors.

Five types of push-out specimens as shown in Figs. 2-4 were prepared. Two samples of each type were tested and the laboratory results averaged. The resulting load versus slip curves are shown in Figs. 5 and 6 while a summary of the push-out results is given in Table 1.

It will be noted that all push-out specimens consisted of two pieces of cold formed steel OWSJ chord sections welded together. The relatively thin chord sections (0.110") were an important aspect of the test program. Goble conducted a series of tests on thin flange push-out specimens with welded stud shear connectors as shown in Fig. 7. It was found that there is a limiting thickness below which the failure mode shifts from stud shear to flange pull out. For 1/4" Ø studs, the limiting thickness is about 0.18 in. However, even when the steel thickness is only 0.110", about 75% of the maximum strength of a Ø stud can be developed as shown in Fig. 7.

In light of this information, and the fact that cold formed chords in joists spanning the economic limit (40') or greater are at least 0.110" thick, 0.110" was chosen as the chord thickness for the tests.

Another noteworthy detail of the push-out specimen is the presence of two open cells in the concrete slab. These cells left a concrete "rib" which contained the shear connectors. Therefore, the push-out test specimens represent a building having open web steel joists, cellular steel floor and shear connectors welded through the steel floor to the top chord of the joist. The concrete in a rib adds appreciably to the strength of a shear connector and, hence, when discussing shear strength, the term "connection" is used. The strength of a "connection" is the combined strength of the concrete rib and the shear connector it contains.

From the load-slip curves, the maximum load sustained by a push-out specimen and its elastic modulus is readily obtained. These values for each of the 5 specimen types are given in Table 1.

Push-out specimen Type I consisted of two connections and hence the specimen values are halved to obtain the maximum strength per connection (q) and the connection elastic modulus (K). Specimen types 2, 3, and 4 consisted of two connections; however, each connection contained two mechanical connectors. For design purposes, it is convenient to define a connection as one concrete rib containing one mechanical connector. Therefore, for these specimens, the maximum test load and the specimen elastic...
2 - 3/4" Riddle Hole (Each Side)

TYPE 2

2 - Hole with Plow Weld (Each Side)

TYPE 3

CIRCULAR PLAIN WASHER:
1 1/8" x 0.125" L.D.

TYPE 4

2 - Washer with Plow Weld (Each Side)

TYPE 5

Note: All other details similar to TYPE 1

FIG. 5 PUSH-OUT SPECIMEN (TYPES 2 & 3)

FIG. 4 PUSH-OUT SPECIMEN (TYPES 4 & 5)

FIG. 1

COST PER FINISHED SQUARE FOOT, DOLLARS

NOTES:
1. Steel chord: 6 - 0.150" L.D.
   f_y = 55,000 psi
2. Stud: 1/2" x 3" long
3. Concrete: f'_c = 3000 psi
4. Deck: 1 1/2" deep, #22 ga.

Concrete slab: 3/8" x 2 1/8", 6" c-c

FRONT ELEVATION

SIDE ELEVATION
Fig. 7: Stud Capacity vs Steel Thickness

(from Ref. 2)

Table 1: Test-Off Results

<table>
<thead>
<tr>
<th>Connector Type</th>
<th>1.1/4&quot; diameter stud</th>
<th>2. 3/8&quot; diameter stud</th>
<th>3. 2.5&quot; fillet weld</th>
<th>4. 1.9/16&quot; plate weld</th>
</tr>
</thead>
<tbody>
<tr>
<td>Per Specimen</td>
<td>Per Specimen</td>
<td>Per Specimen</td>
<td>Per Specimen</td>
<td>Per Specimen</td>
</tr>
<tr>
<td>Load (kips)</td>
<td>Load (kips)</td>
<td>Load (kips)</td>
<td>Load (kips)</td>
<td>Load (kips)</td>
</tr>
<tr>
<td>11.0</td>
<td>11.0</td>
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<td>11.0</td>
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<td>1.00</td>
<td>1.00</td>
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<tr>
<td>0.70</td>
<td>0.70</td>
<td>0.70</td>
<td>0.70</td>
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</tr>
<tr>
<td>5.5</td>
<td>5.5</td>
<td>5.5</td>
<td>5.5</td>
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<td>1.0</td>
</tr>
<tr>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
<td>0.35</td>
</tr>
<tr>
<td>0.06</td>
<td>0.06</td>
<td>0.06</td>
<td>0.06</td>
<td>0.06</td>
</tr>
<tr>
<td>0.28</td>
<td>0.28</td>
<td>0.28</td>
<td>0.28</td>
<td>0.28</td>
</tr>
</tbody>
</table>

Fig. 5: Load-Slip Curves (Types 1, 2 & 4)

Fig. 6: Load-Slip Curves (Types 3 & 5)
modulus is divided by four to obtain \( q \) and \( K \). Since \( q \) and \( K \) determined in this manner are less than the values which would be obtained if each rib in the test specimen had only one mechanical fastener, this approach is conservative.

From Figs. 5 and 6, it can be seen that the load-slip behaviour of the puddle weld and the shear strap is not ideal. Both types of connection tend to reach their maximum load value and then undergo a rapid drop off in load. This type of behaviour is undesirable and these types of connectors need further development.

The load-slip behaviour of the stud, hat and washer with plug weld, however, are fully satisfactory. These three types of connectors all exhibit a good amount of ductility and are able to sustain load through large imposed deformations.

Composite OWSJ Tests:

One of the objects of this paper is to analyze the results obtained from three full size open web steel joist tests. Each test joist was of a different type and they are designated as the Stelco Tower Joist, the CSICC Joist and the Hambro D-500 Joist.

Construction details for the three test joists are given in Figs. 8, 9, and 10 while the test arrangements are given in Figs. 11 and 12.

Stelco Tower Composite Joist

The joist shown in Fig. 8 is currently being fabricated for the 32 storey Stelco Tower in Hamilton, Ontario. The overall composite joist depth is 30" and the typical span is 40". All steel members in the joist are cold-formed from sheet steel and have a minimum yield strength of 55,000 psi after cold-forming. The chords are cold formed hat sections while the webs are cold-formed open seam tubes which are stitch welded.

It can be seen that a 2"-0" wide opening is provided at mid-span for an air conditioning duct. Chord reinforcements were added at this location.

Two types of shear connector were used on the test joist as shown in Fig. 8 - 2" deep hat connectors (see Fig. 3) and 1/4" studs. All connectors were directly welded through the 1/4" cellular steel floor to the underlyong steel chord which was 0.153 in. thick. The total thickness of concrete slab was 4".

As shown in Fig. 11, two point loading was used during the test. Therefore, the bending moment is constant over the 10' long section between the point loads. The critical section under live load bending moment is just outside the ends of the chord reinforcements and is shown as "A-A" on Fig. 11.

CSICC* Joist

This joist test was the first in a series of 5 now being conducted for the CSICC by Dr. Hugh Robinson at McMaster University in Hamilton. The joist members were double angles and bars of CSA Standards G40-12 steel (Fy - 44,000 psi, minimum) and details are given in Fig. 9. The test construction incorporated 1/4" cellular steel floor running continuously over the top chord of the joist. Stud shear connectors, 3/4"O, were directly welded through the steel floor to the top chord of the joist. The test joist spanned 50', was 36" in overall depth, and meets all of Metropolitan Toronto's SEF schools' design criteria.

Two point loading, as shown in Fig. 11, was used in the laboratory test.

Hambro D-500 Joist***

The D-500 composite joist is intended primarily for apartment buildings. Its extreme shallowness makes it ideal for this application. As shown in Fig. 10, the overall joist depth for a 20'-4" span is only 12".

The test joist consisted of a cold-formed steel top chord having 39,200 psi minimum yield strength after cold forming. The yield strength of the two rods (5/8"O and 3/4"O) in the bottom chord was 45,000 psi average.

The vertical leg of the top chord has slots punched into it which permits the use of a unique slab forming system. Roll bars, inserted into the slots, support the plywood formwork and provide lateral support to the top chord of the joist until the concrete hardens. Prior to concrete hardening, the allowable load the top chord may carry is dictated by local buckling. The unsupported vertical leg, with a w/t ratio of 23, governs and results in an allowable stress of 15,300 psi or an allowable force in the top chord of 8.25 kips.

After the concrete has hardened, the top chord is firmly embedded in the concrete and serves as the connecting structural link between the concrete slab and the steel joist.

As shown in Fig. 12, a 3-joist assembly was loaded with concrete blocks to evaluate its composite behaviour.

Test Results

The test results for the Stelco Tower Joist, the CSICC test joist and the D-500 joist are reported in References 4, 5, and 6.

Load-Deflection Curves

Load-Deflection curves are given in Figs. 13, 14, and 15. For all three joists, the load is taken as the externally applied live load and does not include the dead load of the steel joist, steel floor and concrete slab. Fig. 11 defines "W" for the Stelco Tower and CSICC Test Joists, while as shown in Fig. 12, the live load on the Hambro joist was applied by the use of concrete blocks.

In Figs. 13-15, it can be seen that the Load-Deflection curve follows an initial straight line. At some load, the composite interaction between the steel and the concrete begins to break down and the curve starts to deviate from the straight line. The curve continues to rise and eventually levels out as the joist undergoes plastic deformation. All three joists exhibited this same plasticity as evidenced by the large deflections which occurred during the tests.

Both the Stelco Tower and CSICC joists were loaded to failure. Collapse in both cases was due to a web member buckling. The web failures were not due to any inherent weaknesses in either web system; rather, it is the author's opinion that the large deflections near failure gave rise to high secondary stresses which eventually combine with the compressive axial load to cause web buckling.

The fact that the Load-Deflection curves levelled out prior to failure indicates that the ultimate bending strength of the Stelco Tower and CSICC test joists was realized prior to web buckling. Although the Hambro joist was not loaded to the point

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*Canadian Steel Industries Construction Council

**Study of Educational Facilities

***Patented
**Fig. 8** STEEL TOWER TEST JOIST

**Notes:**
1. Jacket symmetrical about 4 except 1/2" of studs 3" kg used for end 15' opposite side.
2. Total connectors - 80 hats, 21 studs.
3. Fy steel bottom chord (avg) = 63.1 K.S.I.
4. 1' concrete = 5.15 K.S.I.

**Fig. 9** CSICC TEST JOIST

**Notes:**
1. Shift 1/4" by chord midway between panel points.
2. Fy steel bottom chord (avg) = 59.2 K.S.I. Fy web (avg) = 51.5 K.S.I.
3. f' concr = 4.65 K.S.I.
4. Connections of each location shown. Stagger studs.
5. Total lifts on joint = 32

**Fig. 10** D-500 TEST JOIST

**Notes:**
1. Fy steel: Top chord = 39.8 K.S.I. (actual) Bottom chord = 45.0 K.S.I. (avg, actual)
2. 6" concrete (28 days, actual) = 4.60 K.S.I.
of collapse, once again, the levelling out of the Load-Deflection curve would indicate that the maximum live load applied during the test is close to the ultimate live load the test assembly could have supported.

A theoretical Load-Deflection curve is also plotted on Figs. 13-15. In all cases, complete interaction between the concrete and steel and elastic behaviour is assumed. For the Stelco Tower and CSICC test joists, the theoretical deflections shown were computed by the method of virtual work and considering the joists as trusses. The concrete slab area was converted to an equivalent steel area and this area was taken as that of the top chord. (The area of the steel portion of the top chord was neglected for reasons to be explained later.) The theoretical deflections shown for the Hambro joist were calculated from bending theory (ACI) with a 10% increase to account for shear deflection. Once again, the area of the steel top chord was neglected.

In all deflection calculations, the full width of the concrete slab was used.

A comparison of the theoretical and actual deflections are given in Table II. No discernable pattern is evident which would indicate that the theoretical methods applied (virtual work and bending theory) are, strictly speaking, applicable to a composite joist. It is felt that more accurate theoretical results can be obtained if composite beam theory which accounts for slip between the concrete and steel (partial interaction) were applied. In any event, it appears that the normal method of calculating joist deflections - bending theory with a 10% shear deflection allowance - gives a reasonable estimation of deflection.

As explained in Reference 7, the presence of open cells causes increased slip between the concrete and steel. Thus, when cellular steel floor is employed (as in the case of the Stelco Tower and CSICC test joists), it is suggested that the deflections calculated by bending theory with a shear allowance, be further increased 10-20% to account for slip.

Measured Strains

Measures strains for the Stelco Tower and CSICC test joists are shown in Figs. 16-19. In Figs. 16 and 18, strain blocks are shown for two conditions - a) when the live load "W" on the composite joist results in a bending moment approximately equal to the working bending moment, and b) the last recorded strains prior to joist failure. In Figs. 17 and 19, Load - Strain curves are presented for the top chord of each joist.

The yield strain for the bottom chords of the Stelco Tower and CSICC joists are 1890 and 1776 in./in. x 10^{-5} respectively. From Figs. 16 a) and 16 a) it can be seen that the measured strains when the joists were under working bending moment were well below the yield strains of the bottom chords. The loads associated with working moment were about 13.64 for the Stelco Tower joist and 10.95K for the CSICC joist. From Figs. 13 and 14, it can be seen that under working conditions, both joists are on the straight line portion of the Load - Deflection curve and are behaving elastically.

From Figs. 16 b) and 16 b), it can be seen that prior to joist collapse, the bottom chord strain of both the Stelco Tower and CSICC test joists was well beyond the yield strain of the steel. The presence of these yield strains is a further indication that the ultimate bending strength of both joists was achieved prior to joist collapse.

In Figs. 17 and 19, it can be seen that the dead load of the steel floor and concrete slab, as well as shrinkage, places compressive strains of -394 and -484 in./in. x 10^{-5} in the top chords of the Stelco Tower and CSICC test joists respectively. As the live load "W" was applied to the test joists, contrary to what might be expected, the top chord compressive strain diminished. In fact, the Stelco Tower top chord went into tension and the CSICC top chord approached zero stress. This behaviour indicates that the top chord is actually working in conjunction with the bottom chord to resist the compressive force in the concrete slab of the composite joist. In practice, the short lever arm between the top chord and the concrete slab, as well as the relatively low level of strain in the top chord, means that once the concrete slab has hardened, the top steel chord of a composite joist can be neglected.

Ultimate Strength Calculation

The ultimate bending strength calculations for the Stelco Tower, CSICC and Hambro composite joists are given in Figs. 20, 21, and 22. Basically, the calculations hinge around the strength of the connection between the concrete and steel. The compressive force in the concrete cannot exceed the crushing strength of the concrete, the tensile force in the steel cannot exceed the yield strength of the steel, and neither the compressive force nor the tensile force can exceed the ultimate strength of the concrete-steel connection. For example, let's consider the Stelco Tower joist. Referring to Figs. 8 and 20, the maximum tensile force the bottom chord of the joist can develop is 130K. From Table I, the ultimate strength of a hat connection is 4.75K, and the ultimate strength of a 1/2" diameter stud connection is 5.9K. On one side of the joist there were 24 hats and the point of zero and maximum moment, and on the other side there were 21 studs. Therefore, the ultimate strength of the concrete-steel connection is 114K on the side containing the hats and 115K on the side containing the studs. Consequently, the force in the steel cannot reach its yield force but is limited to the strength of the steel-concrete connection - 114K. The force in the concrete is equal and opposite to that in the steel. The couple created by the steel force and concrete force is, of course, the theoretical ultimate strength of the joist, in this case, 3276 in. K.

A similar procedure was used for the CSICC and Hambro Joists. In the case of the CSICC joist, the ultimate strength of a 3/4" diameter stud was taken as 14.3K from Reference 8. For the Hambro joist, the steel-concrete connection was assumed fully effective because the top chord is securely embedded in the concrete slab and the concrete slab does not contain open cells.

From Figs. 20, 21 and 22, it can be seen that the maximum error between the theoretical ultimate strength calculated in the above manner and the ultimate strength for the two joists, which were loaded to failure, was 1-1/2%, and for the Hambro joist, which was not loaded to failure, was 7-1/2%. It should be noted that the theoretical moment in all cases is less than the maximum applied moment.

Composite (M3S) Design

The author's design procedure for composite open web steel joists is given in Appendix 4A for the Stelco Tower joist.

As explained under the section on "Measured Strains", the top chord can be neglected under live load conditions. Therefore,
The top chord is selected to satisfy only construction loads. In the case of the Stelco Tower, it was decided to apply the Construction Safety Act and a safety factor of 2 was used.

The selection of a bottom chord and the number of shear connectors is dictated by the combination of live load and dead load. The resulting live plus dead load moment is increased by a load factor for ultimate strength design purposes. In this case, a load factor of 1.7 as outlined in the Plastic Design section of CSA Standard S16.10 was used. The theoretical ultimate moment is calculated and since it exceeds the factored live and dead load moment, the bottom chord and number of shear connectors selected are satisfactory.

The design procedure for the joist webs is identical to that used for any open web steel joist. Live load plus dead load results in the maximum web forces. In this case, CSA Standard S136.11 was followed for the web design.

When calculating the live load deflection, the procedure outlined in the section "Load-Deflection Curves" is followed. The full width of the concrete slab is used and the top chord of the steel joist is ignored when calculating the moment of inertia. The deflection calculated from bending theory is then increased by 1% to account for shear deflection. In this case, a 20% allowance was also made for the effects of slip.

The joist is cambered 1.25 in. to account for dead load deflection and one-quarter of the anticipated live load deflection.

<table>
<thead>
<tr>
<th>JOIST</th>
<th>$\delta''$ (Actual)</th>
<th>$\delta''$ (Virtual Work)</th>
<th>$\delta''$ (Bending Theory x 1.10)</th>
<th>Error1 (Virtual Work)</th>
<th>Error2 (Bending Theory x 1.10)</th>
</tr>
</thead>
<tbody>
<tr>
<td>STELCO TOWER</td>
<td>73.4 x 10^-3</td>
<td>59.6 x 10^-3</td>
<td>55.5 x 10^-3</td>
<td>-19%</td>
<td>-24%</td>
</tr>
<tr>
<td>CSICC</td>
<td>60.0 x 10^-3</td>
<td>63.5 x 10^-3</td>
<td>53.0 x 10^-3</td>
<td>+6%</td>
<td>-12%</td>
</tr>
<tr>
<td>HAMBRIDGE D-500</td>
<td>0.44 x 10^-2</td>
<td>0.45 x 10^-2</td>
<td>0.45 x 10^-2</td>
<td>-2%</td>
<td>-2%</td>
</tr>
</tbody>
</table>

1. Error = Theoretical - Actual

\[\text{b} - \text{mid-span deflection (in.)}\]

**FIG. 15** LOAD DEFLECTION CURVE - D500 JOIST
Conclusions

Composite open web steel joists offer many advantages in building construction. Further, they are highly economical for spans over 40 ft. in length.

The results of three full scale composite joist tests would indicate that such joists can readily be designed by the ultimate strength method. A design procedure employing this method has been given.

REFERENCES


8. Dr. H. Robinson, "Tests of Composite Push-Out Specimens with
APPENDIX A

DESIGN - STELCO TOWER JOIST

Span = 40'0  Fy = 55 K.S.I.  f_c' = 3.0 K.S.I.  S'c-c  
max. L.L. = 1"

D.L. = 45 psf (Concrete, steel floor, GMSJ)
D.L. = 10 psf (Ceiling, mech. & elec.)
L.L. = 45 psf (incl. partitions)

TOTAL 140 psf

Construction = 25 psf L.L. + 45 psf D.L. = 70 psf

\[
M_{D.L.} = (0.65 \times 5 \times 40) \times 12 = 660 \text{ in.k.}
\]

\[
M_{L.L.} = 1020 \text{ in.k.}
\]

TOTAL 1680 in.k.

\[
\frac{A}{4} = 1.492 
\]

\[
r_y = 0.40 \
\frac{I_{xx}}{2} = 0.26
\]

\[
A = 2.08 
\]

\[
r_y = 0.02 
\frac{I_{xx}}{2} = 1.14
\]

Construction Safety Act 54.5 = 2.0  ..  F = 55 = 27.5 K.S.I.

\[
F_a = 1.27 (1 - 0.0035 \times 54) = 22.3 \text{ K.S.I.}
\]

\[
D.L. = L.L.
\]

Top Chord Axial Force  = 840 = 34.0 K  \frac{F_a}{G} = 34.0 = 22.8 \text{ K.S.I.}

\[
24.83 \times 1.492 = 36.03
\]

Panel Spacing = 24"  ..  M panels = 0

\[
K_L = 0.9 (24) = 54 
K_L = 0
\]

\[
S_{136} \quad F_a = \frac{G}{f} = 1.0
\]

\[
F_a = (1-0.0035 \times 54) \times 43 \]

\[
= 28.3 \text{ K.S.I.} \quad 0.65
\]

Top Chord & Connectors

M  D.L. = L.L. = 1680 in.k.

\[
S_{136} \quad L.F. = 1.70 \quad \text{Plastic Design}
\]

\[
M = 1680 \times 1.7 = 2860 \text{ in.k.}
\]

Failure read.

\[
P_y = 55 \times 2.06 = 113
\]

\[
m_y = 114 \quad m_y \text{ stud} = 115
\]

\[
P_C = 113
\]

\[
a = 0.85 \times 5 \times 60 = 0.74
\]

\[
\frac{30 - 1.00 - a/2 = 28.63 \text{ in.}}{M} = 26.63 \times 113 = 3240 \text{ in.k.} \quad 0.65
\]

\[
W_e = 660 
\frac{19}{24} = 0.91
\]

\[
W = 85 \text{ psf}
\]

\[
W = 55 \text{ psf}
\]

\[
25 \times 70 = 1750
\]

\[
D.L. \quad \text{Deflection} 
\]

\[
W = 85 \text{ psf}
\]

\[
W = 55 \text{ psf}
\]

\[
1 = 1440 \text{ in.}^4
\]

\[
\text{Comp.}
\]

\[
\frac{C}{5} = \frac{1}{1.10} \times \frac{1.20}{1.10} = 0.76
\]

\[
D.L. \quad \text{Deflection}
\]

\[
W = 55 \text{ psf}
\]

\[
W = 85 \text{ psf}
\]

\[
1 = 1440 \text{ in.}^4
\]

\[
\text{Comp.}
\]

\[
\frac{C}{5} = \frac{1}{1.10} \times \frac{1.20}{1.10} = 0.76
\]

\[
\text{Steel Joist} = 526
\]

\[
\text{Steel Joist} = 526
\]

\[
\text{Steel Joist} = 526
\]

\[
\text{Steel Joist} = 526
\]

\[
\text{Steel Joist} = 526
\]

\[
\text{Steel Joist} = 526
\]
1. CSICC JOIST UNDER TEST AT McMASTER UNIVERSITY, HAMILTON

2. CSICC JOIST AFTER FAILURE.
   NOTE BUCKLING OF THE DOUBLE ANGLE COMPRESSION WEB.

3. STELCO TOWER JOIST DURING TEST AT McMASTER UNIVERSITY, HAMILTON.

4. STELCO TOWER JOIST AFTER FAILURE.
   NOTE LARGE DEFORMATION AND BUCKLED COMPRESSION WEB MEMBERS.

5. OTTAWA, ONTARIO, APARTMENT BUILDING INCORPORATING HAMRIO B-500 COMPOSITE JOISTS.
   SPECIAL DETAILS HAVE BEEN DEVELOPED FOR THE BALCONY FLOORS.

6. HAMRIO JOIST SHOWING COLD FORMED TOP CHORD CONTAINING "ROLL BAR" SLOTS.

7. APARTMENT FLOOR DURING CONSTRUCTION SHOWING HAMRIO JOIST,
   TEMPORARY "ROLL BARS" AND PLYWOOD FORMING.